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BOSTON SOCIETY OF CIVIL ENGINEERS
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PROCEEDINGS

NOTICE OF REGULAR MEETING.

A REGULAR meeting of the Boston Society of Civil Engineers will be held on

WEDNESDAY, OCTOBER 18, 1916,

at 8 o'clock P.M., in CHIPMAN HALL, TREMONT TEMPLE, BOSTON.

Mr. H. Whittemore Brown will present a paper entitled, "The Groined Arch as a Means of Concrete Floor Construction." It will be illustrated with lantern slides.

S. E. TINKHAM, *Secretary*.

PAPER IN THIS NUMBER.

"The Groined Arch as a Means of Concrete Floor Construction." H. Whittemore Brown.

CURRENT DISCUSSIONS.

Paper.	Author.	Published.	Discussion Closes.
"The Commercial Fertilizer Industry."	L. W. Tucker.	Sept.	Nov. 10
"The Sewage Disposal Problem Confronting Philadelphia."	W. L. Stevenson.	Sept.	Nov. 10

Paper.	Author.	Published.	Discussion Closes.
"Transforming Latitude and Longitude into Plane Co- ordinates."	S. H. Thorndike.	Sept. Nov. 10	

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Contributors are hereby notified that proof will not be submitted to them for examination unless requested before the 10th of the month preceding the month of publication.

MINUTES OF MEETING.

BOSTON, MASS., September 20, 1916. — A regular meeting of the Boston Society of Civil Engineers was held this evening in Chipman Hall, Tremont Temple, and was called to order at 8 o'clock by the Senior Vice-President, Ralph E. Curtis.

There were 94 members and visitors present.

By vote, the record of the June meeting was approved as printed in the September JOURNAL.

The Secretary reported for the Board of Government that it had elected the following to membership in the grades named:

Members — Arthur Leon Ford and Nassime Solomon Klink.

Junior — Benjamin Harrison Snow.

The Secretary further announced that the Board of Government had leased Room 709 in Tremont Temple, adjoining those already used for the Society quarters, at a rental of \$450 per year, and had arranged for the Hersey Manufacturing Company to move to the new room and for the New England Water Works Association to take the room heretofore used by that company, the Society to receive \$300 additional rent on account of the changes made.

Memoirs of deceased members were received and ordered printed in the JOURNAL, as follows: That of Edmund K. Turner, prepared by Past President George F. Swain; that of Samuel Foster Jacques, prepared by William Nelson, F. Herbert Snow and Harrison P. Eddy; and that of Charles Wilson Ross, pre-

pared by William E. McClintock, Edwin H. Rogers and George E. Stuart.

Mr. Eugene E. Pettee then presented a very interesting paper entitled, "The Construction of the Portland Bridge." The paper was illustrated by a large number of lantern slides.

A short discussion followed, and Mr. Pettee answered a number of questions in relation to the construction of the bridge. Adjourned.

S. E. TINKHAM, *Secretary*.

ORGANIZATION OF A NATIONAL ENGINEER RESERVE.

THE War Department has formulated a plan for the organization of a Reserve Corps of Engineers, and issues an invitation to the engineers of the country to apply for commissions in it. Complete information as to the requirements for commissions in the several grades from second lieutenant to major is now available, together with the method of procedure to be followed to secure such commissions.

Information may be secured by addressing the Engineer Officer, U. S. A., Rivers and Harbors, Barristers Hall, Pemberton Square, Boston, Mass.

The following information has been obtained:

No oral or professional examinations will be required, but recommendations of boards will be required in lieu of such examinations. Candidates will submit evidence of their actual employment in corresponding or higher positions in civil life, and references to persons under whom they have been or are employed. The boards will communicate with such persons and with any others that they deem fit, and upon all the evidence submitted or otherwise obtained will base their recommendations and recommend the appropriate grades for which they deem the successful candidates qualified.

Military experience or training in the regular army, volunteers or national guard, or at training camps or educational institutions, will be noted and reported by the board and considered in making the recommendations.

Reserve officers from the following civilian occupations will

be required for the special services of the Corps of Engineers: Bridge engineers, constructing engineers (earth and concrete), constructing engineers (wharves, piers and buildings), electrical engineers (for small plants and power lines), highway engineers, mining engineers (skilled in tunneling and use of explosives), railroad engineers (construction and maintenance), railroad operating officials, sanitary engineers, topographical engineers.

INTERESTING NOTES ABOUT MEMBERS OF THE SOCIETY.*

FOR the benefit of friends, I wish to state that the publication of my obituary in a recent issue of an engineering magazine was a serious mistake. I am thankful to say I am well and very much alive.

FRANK LEMUEL CLAPP.

BOSTON, October 4, 1916.

MR. L. K. ROURKE has been retained to make an investigation and report of a railroad in Chile. He sailed from New York on September 9 and was due at Santiago on October 5.

APPLICATION FOR MEMBERSHIP.

[October 4, 1916.]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications re-

* It is proposed to publish personal notes in the JOURNAL, provided the members show their interest by sending them to the editor.

lating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of twenty (20) days from the date given.

FRASER, CHARLES EDWARD KINGSTON, Peterboro, N. H. (Age 38, b. England.) Graduate of International Correspondence Schools, 1910. From 1901 to 1907, engaged in landscape engineering; from Aug., 1907, to April, 1908, inspector for Engineer Corps, Pennsylvania Tunnels, New York to Long Island City; from April, 1908, to July, 1916, engaged in landscape engineering and construction; from July, 1916, to date, engineer of construction, All Saints Church, Peterboro, N. H.; has executed plans for various architectural firms of Boston and New York; is at present working on plans of Cram & Ferguson, Boston. Refers to G. P. Connolly, S. J. Connolly, T. F. Gray, B. L. Makepeace, J. C. Olmstead and G. F. Webb.

LIST OF MEMBERS.

ADDITIONS.

CHASE, GUY H.....	Fitchburg, Mass.
KLINK, NASSIME S.....	309 Washington St., Quincy, Mass.
NAWN, LEO JOSEPH.....	188 Seaver St., Roxbury, Mass.
SNOW, BENJAMIN H.....	86 Wyllis Ave., Everett, Mass.

CHANGES OF ADDRESS.

BROWN, T. MORRIS.....	38 N. Burnett St., East Orange, N. J.
CARTER, GALE A.....	Box 54, East Brownfield, Me.
COBURN, WILLIAM H.....	Waverley, Mass.
COFFIN, S. P.....	8 Roland St., Charlestown, Mass.
CUNTZ, WILLIAM C.....	120 Broadway, New York, N. Y.
DEMING, GUY S.....	128 East 19th St., New York, N. Y.
GREEN, HOWARD W.....	Y. M. C. A., Elyria, Ohio
HURD, STEPHEN P.....	519 Cutler Bldg., Rochester, N. Y.
MCCORKINDALE, RALPH I.....	32 Laurel St., Fairhaven, Mass.
ROBINSON, ASHLEY Q.....	81 Richardson St., Newton, Mass.
RUNELS, RALPH E.....	783 Main St., Laconia, N. H.
SANDO, WILL J.....	1312 Wells Bldg., Milwaukee, Wis.
SOKOLL, JACOB M.....	32 State St., Cumberland Mills, Me.
TURNER, CHARLES C.....	37 Taylor St., Wollaston, Mass.

DEATH.

SHEDD, FRANK EDSON.....	September 22, 1916
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EMPLOYMENT BUREAU.

THE Board of Government maintains an employment bureau for the Society, to be a medium for securing positions for its members and applicants for membership, and also for furnishing employees to members and others desiring men capable of filling responsible positions.

At the Society rooms two lists are kept on file, one of *positions available* and the other of *men available*, giving in each case detailed information in relation thereto.

MEN AVAILABLE.

No. 372. Age 26. Student for three years at Houghton School, Boston, course in mechanical drawing; is now correspondence school student in mechanical engineering. Desires position as mechanical draftsman or tracer. Salary desired, \$10 per week.

No. 373. Age 32. Educated at Union College and Albany Law School. Has had twelve years' experience with General Electric Co., including computation of costs, prices, discounts, quotations, factory statistics and production; requisition work in order department at New York office; general office work as assistant to manager of sales department and supervision of special manufacturing. Desires position as office manager, with chance for advancement. Salary desired, \$1 800 per year.

No. 374. Age 32. Educated in public schools, normal school and at Boston University. Experience consists of work in civil engineering departments of M. K. T. R. R., G. C. & S. F. R. R., and B. & M. R. R.; dam construction, Pittsfield, Mass.; and work with Crosby Steam Gauge Co., Charlestown, Mass. Desires work on hydraulic problems. Salary desired, \$5 000 per year.

LIBRARY NOTES.

BOOK REVIEW.

EXAMPLES IN MAGNETISM, by Prof. F. E. Austin, Hanover, N. H. 2d ed. 1916. Leather, 5 by 8 in., 90 pages. \$1.10.

Reviewed by Howard S. Knowlton.

The author of this work has produced a clear-cut and interesting handbook of pocket size which is bound to be useful to all students of magnetic problems. The book is printed in green ink on rough paper, so that its use under artificial light can be

enjoyed without the strain upon eyesight frequently induced by glazed paper. It includes much valuable data in regard to symbols, metric units and equivalents, and trigonometric functions, followed by a large number of problems and examples of the application of various magnetic theories. In each case an example is given, with the algebraic or symbolic expression denoting the physical law applying, with the given data systematically tabulated. This is followed by a solution, the process of which is explained at every step. Every solution is then followed by a problem to which the answer but not the detailed working out is given, and the problem is followed by another dealing with some phase of the same subject, to which no answer is given. A vast amount of information upon this important though somewhat specialized subject is included in the ninety pages of this little work, one of the most interesting to the civil engineer being a table of the three variable magnetic elements for the principal states and territorial possessions of the country.

RECENT ADDITIONS TO THE LIBRARY.

U. S. Government Reports.

Anticlinal Fold near Billings, Noble County, Oklahoma.
A. E. Fath.

Antimony Deposits of Alaska. Alfred H. Brooks.

Reconnaissance of Archean Complex of Granite Gorge,
Grand Canyon, Arizona. L. F. Noble and J. Fred. Hunter.

Asphalt, Related Bitumens, and Bituminous Rock in 1915.
John D. Northrop.

Brick Roads. Vernon M. Peirce and Charles H. Moore-
field.

Chisana-White River District, Alaska. Stephen R. Capps.
Gypsum in 1915. Ralph W. Stone.

Lignite Field of Northwestern South Dakota. Dean E.
Winchester and others.

Lode Mining in Quartzburg and Grimes Pass Porphyry Belt,
Boise Basin, Idaho. E. L. Jones.

Manganese and Manganiferous Ores in 1915. D. F. Hewett.
Molybdenite and Nickel Ore in San Diego County, California. F. C. Calkins.

Possibilities of Oil and Gas in North-Central Montana. Eugene Stebinger.

Some Paleozoic Sections in Arizona and Their Correlation. Frederick Leslie Ransome.

Mechanics of Panama Slides. George F. Becker.

Potash Salts, 1915.

Pottery in 1915. Jefferson Middleton.

Public Road Mileage and Revenues in Middle Atlantic States, 1914.

Secondary Metals in 1915. J. P. Dunlop.

Silver, Copper, Lead, and Zinc in Central States in 1915. J. P. Dunlop and B. S. Butler.

Spirit Leveling in Arkansas, 1896 to 1915, inclusive. R. B. Marshall.

Talc and Soapstone in 1915. J. S. Diller.

Triangulation in California, 1913-1915. R. B. Marshall.

Structure of Vicksburg-Jackson Area, Mississippi, with Special Reference to Oil and Gas. Oliver B. Hopkins.

Water-Supply Papers 360, 384, 387 and 399.

Municipal Reports.

Baltimore, Md. Annual Report of Sewerage Commission for 1913.

Baltimore, Md. Annual Report of Water Board for 1914.

Newton, Mass. Annual Report of City Engineer for 1915.

Reading, Pa. Annual Report of Bureau of Water for 1914.

St. Louis, Mo. Annual Report of Water Commissioner for 1915.

St. Paul, Minn. Annual Report of Water Commissioners for 1915.

Salt Lake City, Utah. Annual Reports of City Officers for 1915.

Salt Lake City, Utah. Corporation Appropriation Budget for 1916.

Miscellaneous.

What can Best be Done to Advance the Interests of the Engineering Profession in the United States? J. A. L. Waddell. Gift of author.

Federal Valuation of Common Carriers. James Poyntz Nelson.

Institution of Civil Engineers (London). Minutes of Proceedings, Vol. CCI.

Lehrbuch der Ingenieur- und Maschinen-Mechanik. 4 vols. Julius Weisbach. Gift of E. C. Sherman.

Mechanical Engineers' Handbook. Lionel S. Marks, Ed.

Washington Irrigation Institute: Proceedings of Third Annual Meeting, 1916.

LIBRARY COMMITTEE.

NEW ENGINEERING WORK.

(Under this head a brief description of new engineering work contemplated or under construction will be presented each month. Engineers and contractors are requested to send descriptions of their work to the Secretary, 715 Tremont Temple, Boston, before 1st of each month.)

United States Government. — **NAVY DEPARTMENT.** — *Work Contemplated.* — Magazine buildings of concrete and tile construction, Naval Magazine, Hingham, Mass.; workmen's locker and lavatory rooms, Navy Yard, Boston; sea wall of granite, Naval Hospital, Chelsea, Mass.; 5 000 cu. ft. capacity air compressor, Navy Yard, Boston, Mass.

Commonwealth of Massachusetts. — **METROPOLITAN WATER AND SEWERAGE BOARD.** — *Sewerage Works.* — Work is in progress on the extension of Deer Island sewer outfall, the removal of old Malden River sewer siphon, survey work for Reading extension of Metropolitan sewer, and work on Sections 98, 103, 104, Wellesley Extension.

Bids are advertised for Section 102, Wellesley Extension, and installation of 50 000 000-gal. pumping unit for Ward Street Pumping Station.

METROPOLITAN PARK COMMISSION. — The following work is in progress:

Alewife Brook Parkway. — Surfacing and finishing Alewife Brook Parkway from Massachusetts Ave. to Powder House Boulevard.

Charles River Reservation. — Building reinforced concrete bridge over the Charles River at North Beacon St., Boston and Watertown, A. G. Tomasello, contractor; and reinforced concrete bridge over the Charles River at Commonwealth Ave., Newton and Weston, T. Stuart & Son Company, contractor.

Furnace Brook Parkway. — Surfacing roadway from Quincy Shore Reservation to Hancock St.

New work includes the

Mystic Valley Parkway. — Contract has been awarded to James H. Fannon for grading extension of Mystic Valley Parkway from Mystic St. to Medford St., Arlington.

Old Colony Parkway. — Plans are being prepared for bridge over the Neponset River, Boston and Quincy, to replace old wooden bridge.

Boston Transit Commission. — The Dorchester Tunnel has been practically completed from its beginning at Tremont St. to a point under Dorchester Ave. in South Boston, near Broadway. Work is progressing on the Broadway Station, which extends in Dorchester Ave. from Broadway to West Fourth St. The T. A. Gillespie Co. is the contractor. Work on Section G, which extends in Dorchester Ave. from West Fourth St. to Old Colony Ave., is in progress. Coleman Brothers are the contractors. Section H and Section H Extension, which are completed, extend from Old Colony Ave. to near Dexter St. The next section, Section J, extending from a point near Dexter St. in a southerly direction in and near Dorchester Ave. and Boston St. to near Ralston St., and embracing the station in Andrew Square, is being constructed. The T. A. Gillespie Co. is the contractor.

City of Boston. — PUBLIC WORKS DEPARTMENT, HIGHWAY DIVISION, PAVING SERVICE. — Work is in progress on the following streets:

Condor St.,	Glendon St. to Eagle St.	Sea wall.
Washington St.,	from Morton St. to Asticou Rd.	Granite block.
Bardwell St.,	from South St. to Ballard St.	Asphalt.
Hillcrest St.,	from Elgin St. to Temple St.	Asphalt.
Lorette St.,	from Centre St. to Hillcrest St.	Asphalt.
Temple St.,	from Ivory St. to Spring St.	Asphalt.
Columbia Rd.,	from Edward Everett Sq. to Dudley St.	Granite block.
Everton St.,	from Geneva Ave. to Olney St.	Asphalt.
Normandy St.,	from Intervale St. to Devon St.	Asphalt.
Alexander St.	from Bird St. 527 ft. northerly.	Granite block.
Eric Ave.,	from Savin Hill Ave., northerly.	Asphalt.
Crown Path,	from Claybourne St. to Washington St.	Artificial steps.
Ripley Rd.,	from Harvard St. to Vassar St.	Asphalt.
Harwood St.,	from Willowwood St. to Lucerne St.	Asphalt.
Corbet St.,	from Norfolk St. to Morton St.	Artificial walks.
Dorchester Ave.	from Savin Hill Ave. to Freeport St.	Granite block.
Randall St.,	from Harrison Ave. to Albany St.	Asphalt.
West Canton St.,	from Tremont St. to Appleton St.	Asphalt.
Chandler St.,	from Berkeley St. to Columbus Ave.	Asphalt.
Batterymarch St.,	from Franklin St. to Broad St.	Granite block.
Norway St.,	in front of Christian Science Church.	Artificial walks.

The Fore River Shipbuilding Co., Quincy, Mass., has the following work in progress:

Eleven submarine boats.

Five cargo vessels for the Luckenbach Co., Inc.

One cargo vessel for the Cuba Distilling Co.

Two oil tankers for the Texas Co.

Two tank steamers for the Petroleum Transport Co.

Two stock cargo vessels.

BOSTON SOCIETY OF CIVIL ENGINEERS
FOUNDED 1848

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications

**THE GROINED ARCH AS A MEANS OF CONCRETE
FLOOR CONSTRUCTION.***

By H. WHITEMORE BROWN,† JUNIOR BOSTON SOCIETY OF CIVIL ENGINEERS.

CONTENTS.

GENERAL CONSIDERATIONS.

General outline; plastic nature of concrete; high compressive strength; adaptability of present-day construction to these fundamental characteristics; economy in the reinforced beam; increase in economy gained by the use of an arch; greater compressive strength of arch section; previous use of the groined arch; limitations of use imposed by need of abutments; abutments done away with by use of thrust-resisting rods; other difficulties inherent in arches obviated by use of steel; "beam" reinforcement of other arches not economical; comparatively low rise arches stable under nearly all conditions of loading; each bay made a separate unit; cost of forms not excessive; cost of placing steel reinforcement; salvage on wrecked buildings.

DESIGN OF TEST MODEL.

Need of representative test; methods of analysis; contour of lower face; equations for the moment, shear and thrust at the crown; solution of these equations; design of steel; columns.

NOTE. This paper is published in advance of its presentation before the Society. Discussion is invited, to be received by the editor before December 10, 1916, for publication in a subsequent number of the JOURNAL.

* Abstract of a thesis at the Massachusetts Institute of Technology. June, 1915.

† Assistant, Testing Materials Laboratories, Massachusetts Institute of Technology.

CONSTRUCTION OF TEST MODEL.

Method of loading; use of pig iron in crisscrossed piles; site for test; design of forms; erection of forms; placing the steel; concrete work; modulus of elasticity; deflection apparatus.

TESTING.

Compression tests on cylinders; modulus tests; loading the arch; occurrence of cracks; time deflection; unsymmetrical loading; ease of removal.

CONCLUSION.

LIST OF FIGURES.*

1. Cross-section of reinforced beam.
2. Planes of maximum shear on an arch.
3. Exposed thrust-resisting rods.
4. Enclosed thrust-resisting rods.
5. Pressure lines for different loads on a flat upper surface arch.
6. Pressure lines for loads on an ordinary arch.
7. Method of placing rods making each bay a unit.
8. Plan of groined arch, showing divisions made in analysis.
9. Diagram showing nomenclature of formulæ.
10. Drawing showing graphical work, and the general layout of the arch.
11. Working drawings for forms.
12. Photograph showing arch under load of 450 lbs. per sq. ft.
13. Photograph, load 850 lbs. per sq. ft.
14. Illustrating scale on deflection apparatus.
15. Modulus tests.
16. Deflection of the center of the arch.

GENERAL CONSIDERATIONS.

IN this treatment of the subject of groined arches as a means of concrete floor construction, I shall first trace through the reasons why I believe that present methods of reinforced concrete construction are far from ideal; then show how I believe that a very marked improvement can be made by introducing a groined arch lower face to floors, thereby doing away with all steel reinforcement except a comparatively small amount running along the center lines of the columns to resist the thrust of the arches. Next, I shall treat rather briefly the method employed by me in the design of a test model, and follow this with a description of the construction and testing of this model.

* Figures 1 to 9 inclusive appear on a plate inserted at page 448.

The results of the tests as shown by this description, together with the photographs, I consider ample proof of the statements and reasoning.

Let us first consider two fundamental characteristics of concrete, — its plastic condition while being formed, and the relation between the tensile and compressive strength.

Concrete being mixed and poured on the work, as the custom now generally is, may be molded into any desired shape, the only restriction being upon the containing form and the difficulties encountered in its construction; in other words, as far as the formation of the material itself is concerned, it is readily adapted to a large variety of shapes, including curves as well as angles and straight sections. Compare this with other structural materials, wood, for instance, and we find that wood, if to be made curved, must either be steamed, so that the fibers themselves are bent to conform with the desired curve, or cut through the fibers, in which case the material is subject to a marked weakening. With steel the difficulty again becomes apparent in attempting to roll shapes anything but straight; that is, from the very nature of the manufacture, rolls cannot be easily adjusted to turn out curved members.

Concrete has another marked property which distinguishes it from many other building materials; that is, it is many times stronger in compression than it is in tension. It is the recognition of this property that has given rise to the use of steel reinforcement embedded in the concrete to assist the structure to withstand the tensile stresses.

Next let us consider how present-day reinforced concrete construction makes use of the two characteristic properties of the material above outlined. We find very little attempt being made to make use of the plastic nature of the material. Rectangular beams, etc., are common; in fact, the tendency seems to be rather to adapt concrete to existing types of construction than to develop a type of construction which makes use of the many individual characteristics of the concrete.

Let us analyze the conditions found in an ordinary reinforced beam. Referring to Fig. 1, which shows the distribution of stress over the cross-section of an end-supported beam, it

may be seen that only the material above the neutral axis is under compressive stress; furthermore, this compressive stress varies in magnitude from zero at the neutral axis to the maximum allowable value at the upper surface, thus giving only one half the maximum economy in that portion of the beam which is subjected to compressive stresses. The concrete below the neutral axis carries practically no direct stress, the tensile strength of concrete usually being neglected in the design; this concrete, then, serves only to connect the steel with the concrete above the neutral axis, so that the net economy of the entire section, as far as direct stress is concerned, is less than one third, more nearly one fourth, of its maximum value. Of course the material between the steel and the compression concrete is very necessary in order to carry the shear developed in the beam; however, the shearing strength, or what is more strictly correct, the diagonal tension, is very low when compared with the compressive strength, necessitating the use of stirrups. The economy is increased by the use of a T-section, although the gain is not very considerable.

Let us again turn to our example of wooden construction and see wherein a rectangular section for a wooden beam is an economical section. In the first place, wood is stronger in tension than it is in compression, and also the strength of wood in longitudinal shear is less than one tenth the compressive strength, so that in order to design a section for a wooden beam economically it is necessary to have considerable width to the section at the neutral axis. Again, take the case of the steel I-beam. The flanges are made heavy in order to get a large amount of the material as far from the neutral axis as possible, or where the stresses are the highest. It is not necessary that the web be of very great thickness because of the fact that steel is very nearly as strong in shear as it is in tension.

We are now confronted with the question of how the economy of the concrete can be increased. We must find some form of construction in which the majority of the stresses are compressive, for, as above stated, concrete is considerably stronger in compression than it is in tension. Also, we do not wish to limit ourselves to existing types of construction, but to be free to

choose new types involving curves or other shapes not too complicated for the economical construction of forms.

Arches have been used for several thousand years to span openings by means of masonry construction. Masonry construction usually admits of no tensile stresses; in the case of the arch, the stability of the structure depends upon the stability of the abutment. Then to apply the arch to floor construction in buildings it is necessary to find some form which will have its supports concentrated in four places, thus giving the same columnar form of construction as used in buildings of other types. History again provides us with numerous examples of how this problem may be solved, viz., by the use of the so-called groined arch, which is formed by the intersection at right angles of two barrel vaults, usually having the same rise and span.

Another reason why arches are economical for the use of concrete is that a greater shearing area is obtained because of the increasing thickness of the arch as we approach the abutment, thus increasing the area of the shearing section, as shown on Fig. 2. If C represents the line of action of the resultant compression in the arch ring, the plane of maximum compression will be perpendicular to this line. The plane of maximum shear will be at about forty-five degrees to the plane of maximum compression, as indicated, either ss or $s's'$. Failure along the plane ss will be prevented by the load on the arch; that is, the portion of the arch to the left of the section will tend to slide upward on ss , and this tendency will be prevented, in a large measure, by the load on the floor. Failure on $s's'$ will tend to be prevented by the increased thickness of the arch ring; that is, the area of the plane $s's'$ is greater than that of ss , due to the curvature of the lower face. Thus it might be expected that the ultimate compressive strength of any given cross-section of an arch with a flat upper surface would be slightly, if not considerably, higher than the same area of concrete in direct compression, where the first tendency of failure is in shear, unless specially reinforced to withstand these stresses.

The use of the groined arch is by no means new and novel; the Romans used it to some extent, and medieval builders employed it frequently in their many churches. Its use in con-

crete, however, has been limited to the last decade or two, the first one, so far as I have been able to ascertain, having been built in Ashland, Wis., in 1895, by Mr. William Wheeler, a consulting engineer, of Boston. The arch was here used as a roof of a filter in connection with the water-supply system of the city of Ashland, and supported an earth covering about two feet in thickness. Since that time it may be said that the arch has become the usual form of construction for this type of work, but the extension of its use to other types of structures has been practically nil.

We do not have to look very far to see the reason for this limited use of the groined arch. In the first place, for satisfactory construction of groined arches it is necessary that proper abutments be furnished. In the case of reservoirs and filters, which are usually built below the ground level, this problem is easily solved, because the surrounding earth furnishes an excellent abutment.

When, however, we attempt to apply arches to construction above ground, the problem immediately becomes more difficult. Obviously, abutments in the form of heavy buttresses would be out of the question, both from the point of view of appearance and from economy of construction. We may, however, resist the resultant thrust of the arches by running steel rods from side to side, and end to end, of the building, thus counteracting the resultant thrust at the exterior of the building at its point of emergence. Steel employed this way, however, would have to be exposed throughout the greater part of its length, as illustrated in Fig. 3. As one of the great advantages of a concrete building is its fireproof quality, and as it is a well-recognized fact that exposed steel is susceptible to a marked weakening even by a small fire, this type of construction would practically be ruled out of use. If, however, the resultant thrust is not counteracted directly at its point of emergence, but by means of a direct stress together with a bending moment produced in the column, the rods may be run in the concrete throughout the building, say, about two inches from the surface of the floor, as illustrated in Fig. 4, thus counteracting the thrust without exposing the rods, but producing a bending moment which must be considered in the design of the columns.

The maximum magnitude of this moment is readily computed by statics, and in this case occurs at the resultant thrust with a magnitude of $A \times a$ or $B \times b$ inch-pounds (if the units are given as inches and pounds).

By the use of steel in counteracting the thrust of the arches we obviate one of the serious inherent difficulties met with in their construction, viz., movement of abutments. In this type of construction we have the arch and its abutment integral so that there is no opportunity for any undue stresses from movements of the abutment. Again, by the use of the thrust-resisting steel we also obviate another difficulty met with in arch construction, namely, stresses due to temperature changes in the arch ring. The arch ring and the thrust-resisting steel expanding practically at the same rate make the temperature stresses negligible.

The following extract from Turneaure and Maurer, "Reinforced Concrete Construction," pages 333, 334, gives a good explanation of the necessity of steel reinforcement in bridge arches:—

"Theoretically, the gain in economy by the use of steel in a concrete arch is not great. If the pressure line does not depart from the middle third, the steel reinforces only in compression, and in this respect is not as economical as concrete. If the line of pressure deviates farther from the center, resulting in tensile stresses in the steel, the conditions are such that those stresses must be provided for by the use of steel at very low working values. That is to say, the direct compression in the arch is so large a factor that the limiting stresses in the concrete will always result in very small unit tensile stresses in the steel when any tension exists at all.

"Practically, the value of any reinforcement is very considerable. It renders an arch a much more reliable structure, it greatly aids in preventing cracks due to any slight settlement, and by furnishing a form of construction of greater reliability makes possible the use of working stresses considerably higher than is usual in plain masonry."

Also from Taylor and Thompson, "Concrete, Plain and Reinforced," 1912 edition, page 535:

"We find the use of steel reinforcement in a concrete arch is desirable but not absolutely necessary, as it is possible to

construct an arch like the Walnut Lane Bridge in Philadelphia with the concrete laid in blocks, each block forming a voussoir like the stones in a masonry arch. At the same time, under ordinary conditions, while the introduction of steel does not, with the present knowledge of concrete arch design, permit great diminution in section, it does give considerable added strength at comparatively low cost and may prevent the formation of cracks in the concrete and take tension caused by any unforeseen action of the arch, such as settlement of foundation, improper allowance of temperature, or shrinkage of the concrete while hardening."

It will be seen from the above quotations that the chief purpose of reinforcement in an ordinary bridge arch is not to take care of the principal stresses, as in the case of a beam, but to increase the reliability, allowing for stresses caused by movement of abutments or changes in temperature. It may be said, then, that in general the reliability of an arch is measured by the reliability of its abutments. By taking up the thrust of the groined arch by means of rods, as above indicated, the arch and its abutment are thereby made integral parts of the same structure, and any unreliability of the abutments is entirely obviated. It would seem, therefore, that this form of construction obviated the necessity of any "beam" reinforcement of the arch, the thrust-resisting steel being all that is required.

Next let us consider the stability of the groined arch under various loading conditions.

It is a well-known fact that, in general, arches are better adapted for carrying uniformly distributed loads than concentrated or non-uniform loads. An arch of large rise in relation to span would be unstable under a system of concentrated loads by reason of the fact that the line of pressure in the arch ring would tend to fall without the middle third. However, if we make the arch more nearly flat, that is, make the ratio of the rise to span larger, we obviate this difficulty in a great measure. In a comparatively flat arch the crown thrust, of course, is great, but the arch is very stable under all conditions of loading. This reasoning may be better understood by reference to Fig. 5, which represents a series of concentrated loads upon an arch. If we neglect the dead weight of the arch itself, the line of pres-

sure for the load represented by A would be simply two straight lines from the abutments to the point of loading. Now it will be noticed that this line of pressure lies entirely within the arch ring, whereas if we had an arch of comparatively high rise, such as is represented by Fig. 6, a load at A' would have a line of pressure as represented, but this line would fall without the arch ring a distance represented by d , causing failure of the arch at the haunch. This same advantage of the comparatively flat arch is obvious for loads at B and C , Fig. 5, for the same reason. It will be seen, therefore, that by making the arches comparatively flat, we not only decrease the quantity of concrete, and thereby the dead weight of the building on the foundations, but we also make the arch more reliable from the loading point of view. However, as the thrust of a low rise arch is greater than that of a high rise arch (where the other conditions are the same), this increase in thrust requires the use of more steel to counteract it.

Next let us consider the effect of loading only alternate bays, in a building several bays wide. By reference to Fig. 7, which shows the cross-section of a building of four or five bays loaded only in alternate bays, we see that the arch in bay A would tend to be unstable were the thrust-resisting rods simply run from end to end of the building, because the resulting thrust from arch B would have to be counteracted by arch A , and unless it were loaded it would tend to buckle up. To get around this difficulty and make each one of the bays a unit so that the load on the bay will not affect other bays, the rods should be bent down into each one of the intermediate columns as shown in Fig. 7, thus counteracting the thrust of the bay within itself, but producing a bending moment in the intermediate columns when alternate bays are loaded. In this manner we remove one of the most serious difficulties encountered when a series of arches are subjected to non-uniform loading.

The most important consideration from the practical standpoint is always that of cost. Let us turn, therefore, from the problem of design and compare the cost of the groined arch floor with other existing types.

One of the most important items of expense in the construc-

tion of any concrete building is that of forms. Forms may cost as much as the concrete that is put into them, so that it is imperative that the forms should be simple and easy to erect and handle. In a recent example of groined arch construction in reservoir roofs, wooden forms were used, being made up in convenient sections at a woodworking mill and transported to the job at an initial cost of about twenty cents per square foot. Having them carefully built enabled them to be used seven or eight times with a minimum amount of work in removing and resetting, the total average cost per square foot of the entire roof being given by the contractors as four and one-half cents (Springfield, Mass., Water Supply, The Charles R. Gow Co., contractor). The cost of form work for floors in the Carter's Ink Company factory, a good example of beam and slab construction, was eleven cents per square foot (Aberthaw Construction Co., contractor). The cost in the Winchester Repeating Arms Company factory, a typical "mushroom" or flat slab construction, was reported as twelve and one-half cents per square foot. To be sure, some allowances ought to be made for the increase in cost due to the height of the buildings, but this should dispel any fear that the introduction of curved forms materially increases the cost of form work; the ease of erection and the doing away with large amounts of shoring more than make up for any increase in the cost of the forms themselves.

Next consider the cost of placing the steel reinforcement. In the arch type, steel is only required running from column to column in both directions and has only simple, right angle bends. The amount of steel required is about equivalent, in the case of a test floor, to that required for this size of floor in beam and slab construction. We have, then, with the use of an arch type of floor, an elimination of nearly all of the labor of placing intricate steel reinforcement.

Another distinct advantage in this form of concrete floor construction is found in the fact that when it is desired to tear down a building, the process of wrecking is a relatively simple one, in that there is very little steel used, and what steel is found in the floors is not bonded to the concrete, inasmuch as the steel is in tension and the concrete in compression, so that the two are

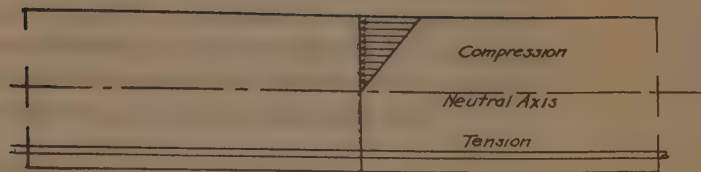


Fig. 1.

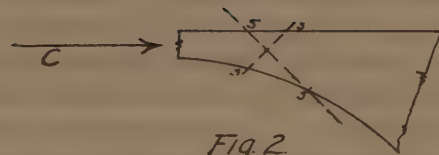


Fig. 2.

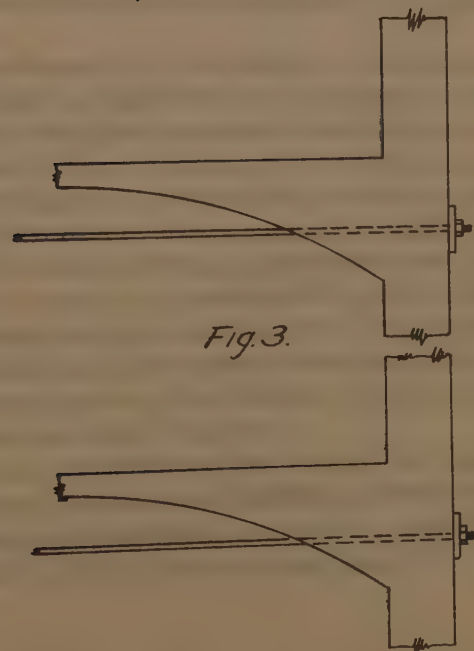


Fig. 3.

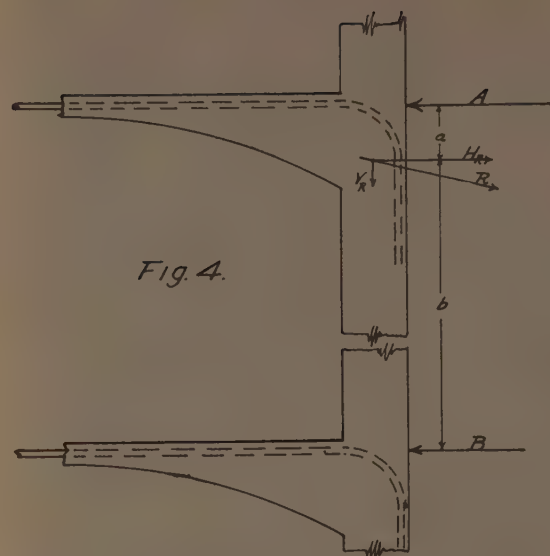


Fig. 4.

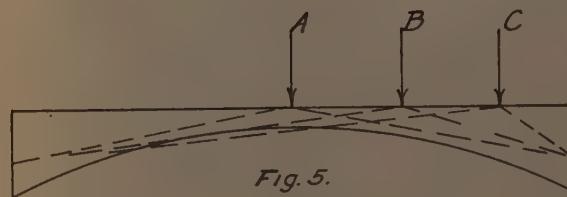


Fig. 5.

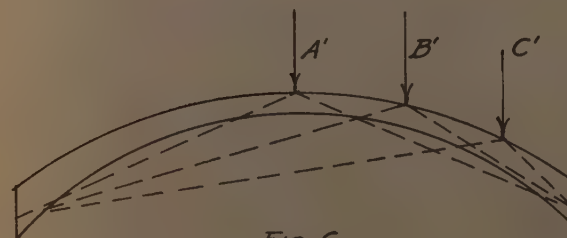


Fig. 6.

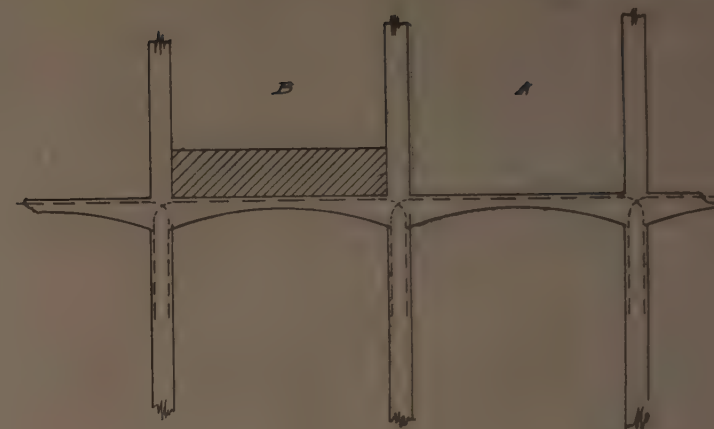


Fig. 7.

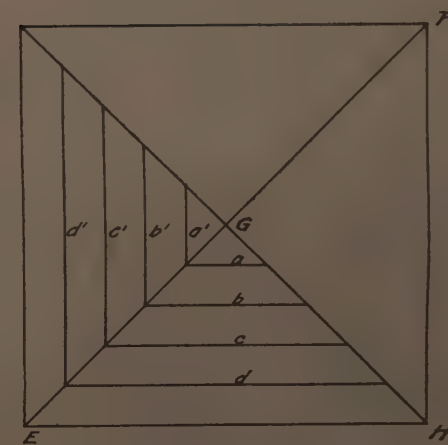


Fig. 8.

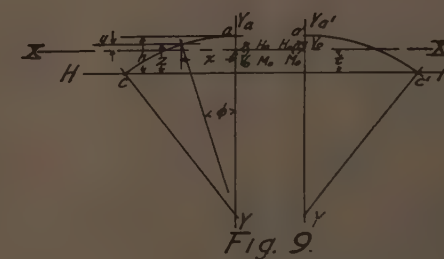


Fig. 9.

readily separated. The salvage from this type of building would be, therefore, quite considerable, as the concrete could easily be crushed for use again as coarse aggregate, and a large part of the steel would also be valuable. Compare this with the present methods of wrecking concrete where the building is a total loss, as all of the material has to be thrown on the junk pile.

DESIGN OF TEST MODEL.

The test of any such idea as here proposed may be divided into two parts, that made by applying known methods of design to several cases of this type of structure, which may be termed a more or less theoretical test, and by the construction of an actual model and testing it to failure. Inasmuch as small models are apt to give results which materially differ from the actual conditions met with in practical construction, the writer has attempted in this test to construct a model of sufficient dimensions to warrant its being recognized as representative. A model which has a clear span of seven feet and is placed on columns four feet long ought to be considered as giving these representative conditions, yet is not without the bounds of reason when considered as an experiment. These were the dimensions adopted for the model constructed.

The subject of the design of groined arches is very sparsely treated in the text-books; in fact, the only treatment of the subject in English which the writer was able to find was the section in a book by Prof. William Cain, entitled "Steel Concrete Arches and Vaulted Structures." Briefly, his method is this: he first considers the arch as cut into a number of slices, indicated in Fig. 8 by *a*, *b*, *c* and *d*. These separate slices are then considered as ordinary arches, and their stability investigated. With the resultant thrust on the groin line *EF* of these several arches as loads, the groin line itself is then investigated for stability, and the resultant thrust at the abutment considered as the resultant thrust of the whole structure.

Another method, recommended by Mr. Horace H. Chase,* who has had experience in designing and building groined arches for reservoirs, as giving safe results, is to consider the whole

* Principal Assistant, F. A. Barbour, Consulting Engineer, Boston.

section *FGH* as an ordinary arch with a uniformly varying load and varying length of cross-section, and then apply the established methods of arch design in this case. Either of the above methods is more or less approximate, but the latter was chosen because it seemed to lend itself a little more readily to solution than that of Professor Cain.

Having decided upon a method of analysis, the next thing to determine is the proper contour to give the lower face of the arch. For a uniform load, the equilibrium polygon is a parabola, and for a load concentrated at the center it is two straight lines from the load to the supports. For a mean between the two, which is approximately the case with a groined arch, the making of the lower face in the form of a segment of a circle would seem to give the line connecting the centers of the cross-sections of the arch thus constructed (segmental lower face and straight upper face) very nearly correct. This form was adopted, and, as will be shown later, gave a line of pressure which diverged but slightly from the centers of the sections. The layout of the arch is shown by Fig. 10.

Having given the span of the arch, it is next necessary to choose the crown thickness and rise necessary to withstand the live load, taken in this case as 150 lbs. per sq. ft. For the first trial a crown thickness of two inches and a rise of twelve inches was taken.

This arch was analyzed according to a method given by Prof. C. M. Spofford in the "Theory of Structures" (Supplement, 1915).

The following equations were obtained by the method of work for the moment, thrust and shear at the crown:

$$M_o = \frac{\int_0^{\frac{s}{2}} (m_L + m_R) \frac{ds}{EI}}{2 \int_0^{\frac{s}{2}} \frac{ds}{EI}} \quad (1)$$

$$H_o = \frac{-\int_0^{\frac{s}{2}} (m_L + m_R) \frac{y ds}{EI}}{2 \int_0^{\frac{s}{2}} \frac{y^2 ds}{EI} + 2 \int_0^{\frac{s}{2}} \frac{ds}{EA}} \quad (2)$$

$$V_o = \frac{\int_0^{\frac{s}{2}} (m_L - m_R) \frac{x ds}{EI}}{2 \int_0^{\frac{s}{2}} x^2 \frac{ds}{EI}} \quad (3)$$

By reference to Fig. 9 the nomenclature of the above formulæ may be more easily understood.

Let: — XX and YY be two axes of reference, the former being so located that $\int_0^s \frac{y ds}{EI} = 0$ and the latter being an axis of symmetry.

x and y be the ordinates of any point on the arch axis referred to these axes.

a be the length of any portion of the arch axis.

M_o be the moment about an axis normal to the paper and passing through the intersection of XX and YY .

H_o , V_o be horizontal and vertical forces respectively assumed as acting on each half of the arch at the intersection of XX and YY .

m_1 be the numerical value of the moment of the applied loads about any point xy on the left half of the arch, considering this half to act as a cantilever beam fixed at the abutment.

m_r be a similar moment on the right half of the arch.

M_c be the actual bending moment about an axis normal to the arch axis at the point a .

$$M_c = M_o - H_o (h - t).$$

I be the moment of inertia of a normal cross-section about an axis normal to the arch axis at xy .

A be the area of a normal cross-section of the arch at the point xy .

E be the modulus of elasticity of the material, which may be considered constant and therefore cancelled from the equations when substitutions are made.

Inasmuch as it is not the object of this paper to present any new theories of arch design, but rather to apply existing methods of ordinary arch design to the case of the groined arch as above outlined, with due allowances for safety required by the lack of knowledge of the subject, I will spend but little time here with computations, giving rather the general conclusion drawn from the test as a whole.

From the computation of the arch as above outlined, the value for the horizontal component of the crown thrust was

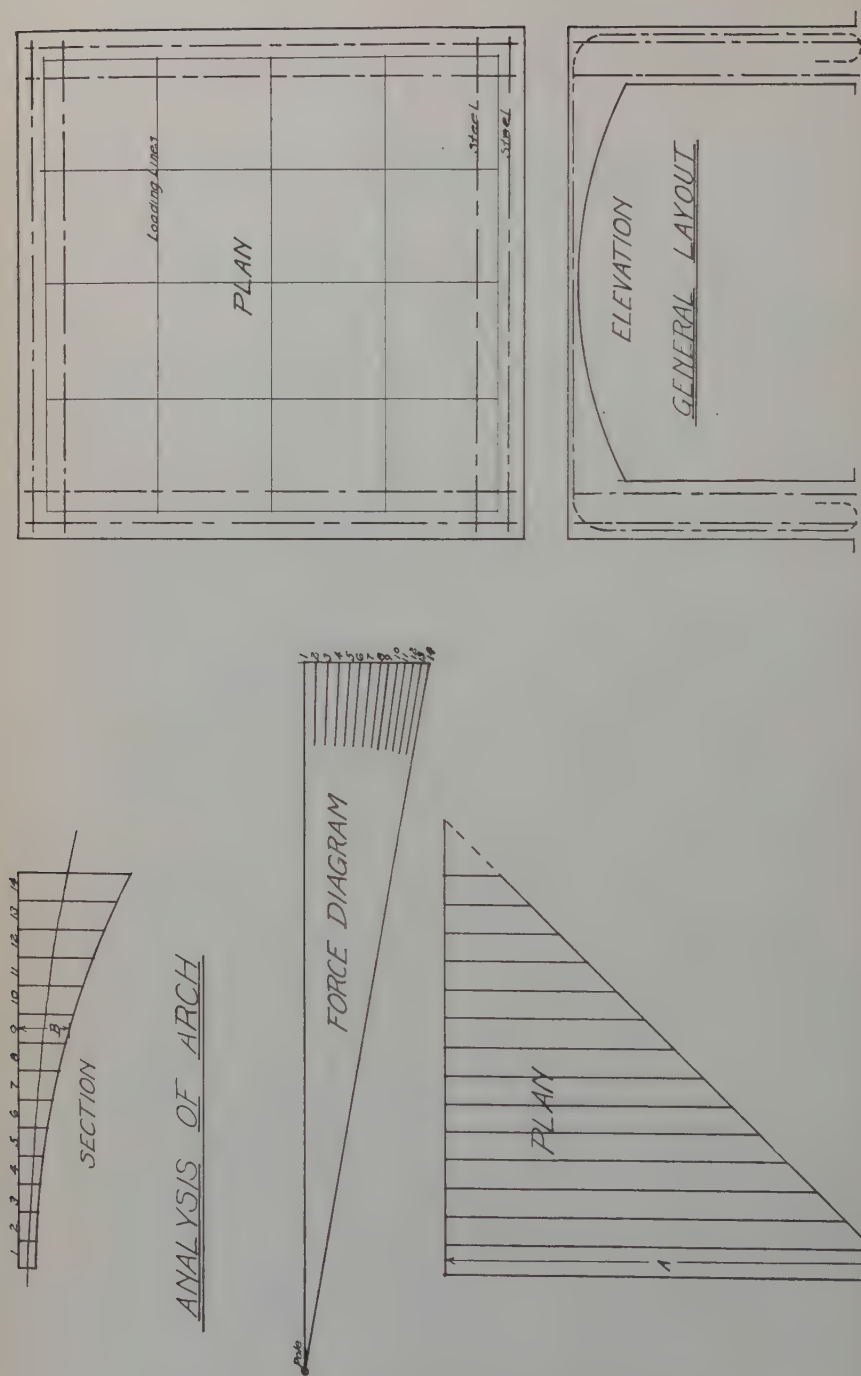


FIG. 10. GRAPHICAL WORK AND GENERAL LAYOUT OF ARCH.

found to be 4 300 lbs. It was assumed that the horizontal component of the resultant thrust was equal to the horizontal component of the crown thrust.

It was decided to use two $\frac{3}{4}$ -in. plain round bars, area 0.88 sq. in. Tests for the yield point showed that it occurred at a stress of 35 000 lbs. per sq. in. With area 0.88 sq. in. and yield at 35 000 lbs. per sq. in., the total thrust at the yield point would be 30 800 lbs. Now as serious cracks did not begin to occur until a load of 750 lbs. per sq. ft. was applied, it would seem reasonable to assume that the steel reached its yield point around the 750 lb. load. I regret that the extensometer readings on the steel did not prove trustworthy enough to allow me to place more reliance on this assumption. On this basis the crown thrust of 30 800 lbs. occurring at a load of 750 lbs. per sq. ft. would mean that the thrust was 7.1 times the thrust at the designed load of 150 lbs. per sq. ft., while the load was only 5 times as great, indicating that either the steel had not yet reached its yield point, that there was a serious error in the assumptions of the design, or that the elastic abutments increased the thrust (no allowance being made for this in the original design). The determination of cause of this discrepancy is the object of a second series of tests which I have under way at the present time, although the actual test has not progressed far enough to allow any conclusions to be drawn.

CONSTRUCTION OF THE TEST MODEL.

In any test it is most essential for satisfactory results that the method of loading should correspond very closely with the method assumed in the theoretical design. In making tests of floors heretofore the loading has been accomplished either by using sand or cement in bags, or pig iron. Any of these methods makes it difficult to obtain room for extensometer readings on the upper surface of the floor, and the upper surface of the floor, in the case of the groined arch, is the one required for these readings on account of the difficulties encountered in attempting to make readings on the curved lower face. Also, all of these methods are likely to have some arch action in the load itself, thus departing very widely from the assumptions of the design.

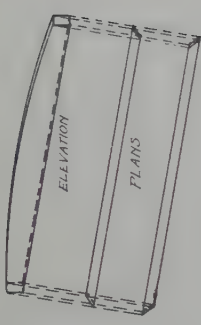
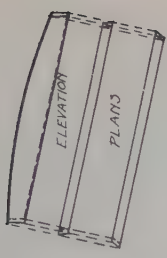
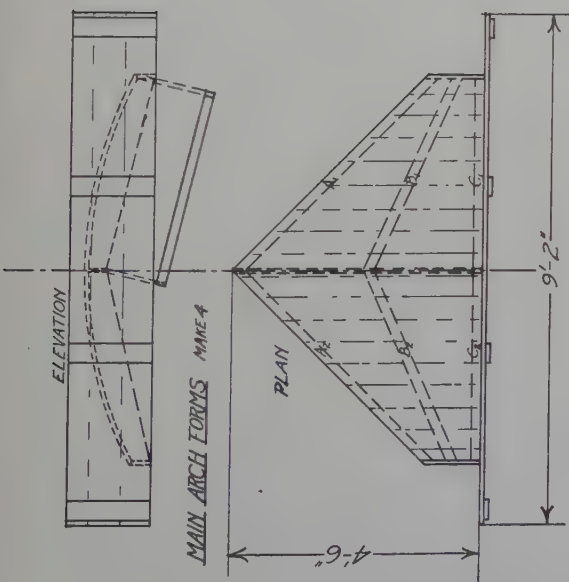
In order to obviate all arch action in the load, and also to allow for extensometer readings on the upper surface, the method of loading used in this test was essentially as follows: Half pigs of iron, as usually found ready for the cupola at foundries, run a little less than twenty-four inches in length and of such width that three laid side by side will occupy slightly less than twenty-four inches. Therefore, by laying out twenty-four-inch squares over the surface of the floor, and planning for extensometer readings along the dividing lines of these squares, the iron may be piled in crisscrossed layers averaging three pigs to the layer, thus making each pile self-supporting and still leaving room for extensometer readings between the piles. (See Figs. 10 and 12.)

As the method of loading as above outlined requires a large amount of pig iron, it was thought advisable to make the test model at or near a foundry where plenty of iron was available. The Waltham Foundry Company of Waltham, Mass., had a little spare room in their yard which they were kind enough to allow me the use of for a few months, and they also had plenty of pig iron handy for use in loading the arch.

A small test pit about two feet deep was made where it had been decided to build the model. The foundation was found to be of coarse gravel, which has a fairly high bearing capacity, at least three tons per square foot. Under a load of four times the designed load, the bearing at the foot of each column was approximately this amount.

The next step was to design the forms. The main arch form was built in four sections which were bolted together along the groin lines, nailed securely at the corners, and finally strips of 2 by 3 spruce were fastened to all four sides and securely bolted at their intersections. All of this was done in order that the arch form should be self-supporting, being, in fact, an arch itself with the thrust counteracted by securely fastening the four sections together. This form rested directly on the tops of the column forms. A good idea of the forms used may be obtained from Fig. 11.

It is interesting to note in the design of the column forms the manner in which the corner joints were lapped in order that the stress on the nails in every case should be shear rather than



FORM DETAILS

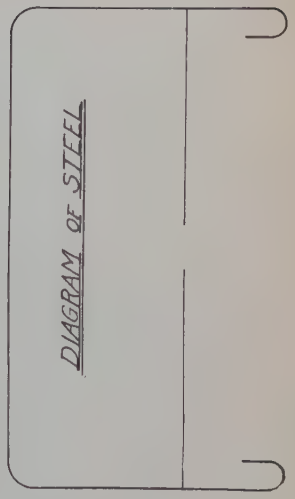
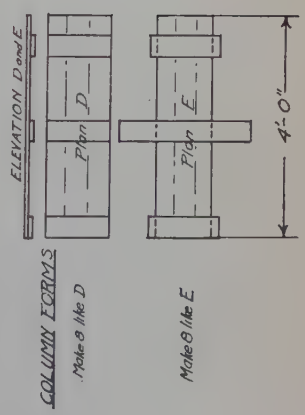


FIG. II. WORKING DRAWINGS FOR FORMS.

straight tension. In this way any tendency towards springing the joints was avoided, and the use of clamps made unnecessary. The forms were made up in the carpenter shop of the Massachusetts Institute of Technology, and transported by truck to Waltham.

The erection of the forms required only one hour's time for the writer, with the aid of a laborer. First, two 3 by 4 spruce joists were laid across each pit for the columns, then the column forms were placed, resting on the joist by the long middle battens. After the column forms were placed and leveled, one of the sections of the arch was placed on top of the column forms and its pointed end held up in place temporarily by a light stick; the next section was similarly placed and then three bolts inserted along the intersection. This process was continued until all four sections were in place; the corners were then securely nailed, and finally, the pieces of 2 by 3 spruce were run around the outside and securely bolted at the corners.

Although some excess cost was entailed by the careful construction of the forms, it was more than made up for by the ease in erection and stripping, and also by the excellent condition of the forms after removal. With very little repairing the forms could easily be used eight or ten times, showing that although the original cost of arch forms may be slightly greater than those for flat slabs, the better construction necessitated more than pays for itself in the time saved in the erection and removal.

The labor necessary in placing the steel was of course very small. As specified in the design, $\frac{3}{4}$ -in. plain round, mild steel bars were used. Tension and bending tests showed material to be about up to the standard, although not what would be termed first-class. Eight rods twenty feet long were ordered; these were bent on the job conforming approximately with the diagram in Fig. 11. Wooden blocks were wired to the bars where it was desired to take extensometer readings.

In placing the steel, care was taken to see that the rods were accurately placed, especially through the crown of the arch and the upper portion of the columns.

The concrete work involved numerous difficulties because of the lack of both time and space. Fairly representative tests,

however, were made of the sand and cement with the result shown by Table I, which shows the materials to be of good average strength.

TABLE I.

CEMENT AND SAND TESTS.

24-hour Neat	
(1) 340	
(2) 390	Av. = 365 lbs. per sq. in.
7-day Neat	
(1) 610	
(2) 585	Av. = 597.5 lbs. per sq. in.
Standard Mortar	
(1) 280	
(2) 300	Av. = 287 lbs. per sq. in.
(3) 280	
Waltham Mortar	
(1) 270	
(2) 295	Av. = 262 lbs. per sq. in.
(3) 270	= 91.5% of Standard.
35-day Neat	
(1) 695	
(2) 725	Av. = 710 lbs. per sq. in.
Standard.	
(1) 290	
(2) 295	Av. = 295 lbs. per sq. in.
(3) 300	
Waltham.	
(1) 290	
(2) 280	Av. = 285 lbs. per sq. in.
(3) 285	= 96.5% of Standard.

Test of the cement with hydrochloric acid showed no signs of dirt or other foreign matter.

RESULTS OF SCREENING TEST OF SAND.

Left on No. 10.....	18.4%
No. 20.....	39.4%
No. 30.....	26.0%
No. 50.....	12.8%
Through No. 50.....	3.0%

This shows a well-graded sand with a minimum of fine material.

Washing the sand revealed no objectionable foreign matter.

In proportioning the mixture to be used, there was little to be considered, inasmuch as it was not the concretè that was being tested, but rather its action in the arch, so that a 1 : 2 : 4 mixture was chosen rather arbitrarily as being of common proportions. Gravel passing a $\frac{3}{4}$ -in. and held on a $\frac{1}{4}$ -in. screen was used. Each batch was made fairly dry, so that in placing the concrete in the arch, the joint between batches could be made nearly perpendicular to the surface of the forms, thus making the joint more efficient in carrying the stress. About an hour after the pouring had been completed, holes were made in the upper surface of the floor by pressing a steel rod into the slightly hardened concrete, thus allowing for the insertion of steel plugs for the extensometers. About twenty-four hours after the pouring, the whole floor was covered with hay and thoroughly soaked, in order that the drying out should not be too rapid. The forms were removed at the end of three weeks.

Extensometer inserts consisting of pieces of $\frac{5}{8}$ -in. steel rods about an inch long were placed in the holes made in the concrete with a mixture of cement and plaster of Paris. After the concrete had been poured it was thought advisable to attempt to make strain gage readings on the concrete in the columns. Plugs were here stuck on the columns with a mixture of cement and plaster of Paris; some of these proved satisfactory and gave readings which checked up closely with the design, while others failed to adhere at all.

In order that the strain gage readings should be of some value, it was necessary to determine the modulus of elasticity of the actual concrete used in the model at the age of testing. For determining the modulus, blocks 8 by 8 by 24 ins. were made of the actual concrete used; $\frac{3}{4}$ -in. rods were placed 20 ins. apart in each block, and held in place by projecting into the sides of the form. As comment on this method of placing the extensometer inserts, it may be said that difficulty was encountered because of the shrinkage of the concrete, which tended to crack the block through the plane of the rods. Since that time I have had opportunity to make other modulus tests on plain blocks, the inserts being made by boring 1-in. holes with a diamond drill,

and setting the steel plugs with neat cement and plaster of Paris.

In order to measure the deflection of the arch it was necessary to design some form of apparatus which should combine the three following elements: First, it should not be elaborate or expensive, because it is likely to be ruined with the failure of the arch; second, it should be easily readable from the exterior of



FIG. 12. LOAD 450 LBS. PER SQ. FT.

the arch, so as not to endanger the observer upon a sudden failure of the arch; and third, it should be fairly accurate, at least to 0.01 in.

It was decided to make the apparatus of wood, mounting a lever with arms as 1 to 50 on a board back, having the fulcrum simply a snug fit about the supporting screw, but having the point of application on the short arm fixed by a steel point fitting into a prick-punch hole, also in steel. A general idea of the whole

apparatus may be gained by reference to Fig. 12. The apparatus was made easily in a few hours by the writer in the Institute carpenter shop.

After construction, it was necessary to calibrate the apparatus. This was accomplished by placing it in a small Riehle



FIG. 13. LOAD 850 LBS. PER SQ. FT.

testing machine, where the deflection could be controlled easily. The actual deflection was measured with a micrometer, and compared with the scale reading on the apparatus, the scale being made with divisions fifty times the actual deflection and placed at a radius distance of fifty inches from the fulcrum. An

error averaging about seven per cent. was found. The radius distance of the scale was then increased $3\frac{1}{2}$ ins., and the readings then were found to be accurate within a few thousandths of an inch. A new scale was then constructed, giving a wider range of deflection, and then tested. In this it was found that the effect of curvature became apparent in the larger deflections.

Here it occurred to the writer that the effect of curvature might be overcome by the use of a scale made of cross-section paper, and reading from a fixed point on the pointer.

From Fig. 14 it may be seen that the deflection recorded on a curved scale *B* corresponds to the deflection of the arc d rather than the actual deflection d_1 ; by the use of a scale as indicated by *C*, the deflection read along the line *c*, the path of a given

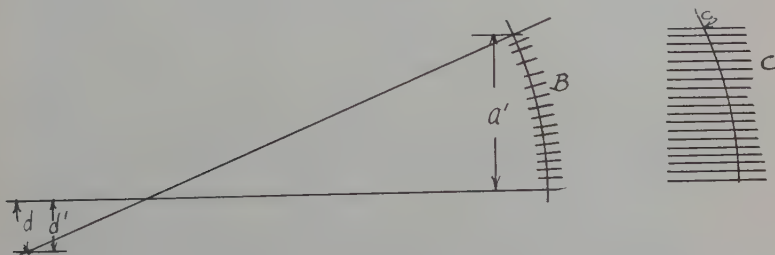


FIG. 14. ILLUSTRATING SCALE ON DEFLECTION APPARATUS.

point on the long arm of the lever, will give the vertical distance a' , which corresponds to the actual deflection d' .

TESTING.

Tests of 6-in. by 6-in. cylinders which had been made as the mixing progressed were made at 7, 14 and 28 days; a strength of 2 070 lbs. per sq. in. being obtained in 28 days.

Modulus tests were made at 21 and 28 days, the results of which are plotted in Fig. 15. The average of the 28-day tests used in the checking of the design was 1 178 000 lbs. per sq. in.

Before commencing the loading of the test floor it was necessary to mark off the squares for the piles of pig iron, so that the loading might be carried on expeditiously without interfering with the extensometer inserts. The inserts were then

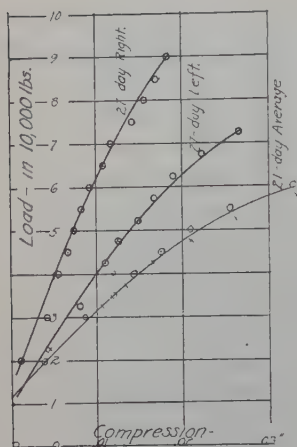


FIG. 15. MODULUS TESTS.
LOAD-COMPRESSION
CURVES.

punched and bored for a gage length of twenty inches, a 20-in. Berry strain gage being used.

In order that there should be a minimum amount of lifting in connection with the handling of the iron, a one-horse cart was procured, so that it was only necessary to load the iron into the cart from the pile, weigh the load, and then back the cart up to the floor, which was purposely made of convenient height for this operation. In this manner it was possible to carry on the loading quite rapidly. Loads were applied in increments of 1 600 lbs., equivalent to 25 lbs. per sq. ft., until the designed load was

reached, and after that in increments of 3 200 lbs. until a load of 54 400 lbs. had been applied, equivalent to 850 lbs. per sq. ft.

Extensometer readings were taken after adding each increment of load. Dust and dirt made it difficult always to be sure that the holes for the point of the extensometer were clear, which might account for the sudden variations in some of the readings.

After a load of 150 lbs. per sq. ft. was reached, the readings on the interior of the arch had to be abandoned on account of the increasing height of the piles of iron.

Deflection readings were also taken with each increment of loading. (Fig. 16.) After a load had been applied and before another was ready, some slight settlement occurred, usually not over .003 or

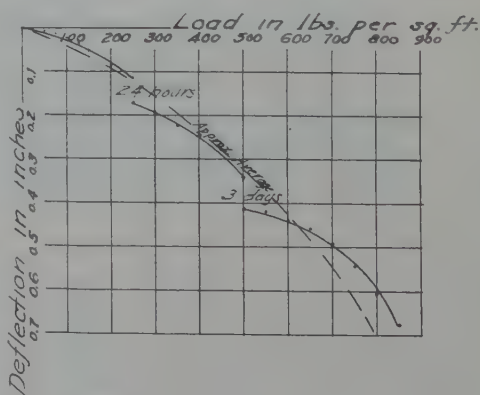


FIG. 16. DEFLECTION OF ARCH.

.004 in. Twenty-four hours elapsed before load 250 and 300, and during this time a settlement of 0.06 in. occurred. The total deflection immediately after the application of the 500 load was 0.342 in. Three days elapsed between load 500 and 550, during which time a deflection of 0.069 in. occurred, showing that the deflection becomes practically constant after twenty-four hours. The total deflection upon the application of the 850 lb. load was 0.675 in. It will be noted from the photographs that the deflection apparatus rested directly on the ground; thus the deflection readings included not only deflection of the arch but also the settlement of the whole structure on its foundations.

When a load of 750 lbs. per sq. ft. was reached, cracks were noticed on the under side of the concrete at the crown and on the outside of the columns about a foot from their tops. The next increment of loading opened these cracks and produced another set running diagonally across corners of the top of the arch about two feet from the columns. Explanation of this would seem to be that the steel had reached its yield point, thus allowing the center of the arch to fall slightly, causing cracks at the lower side at the center, on the upper side near the ends and on the outside of the columns where the tension caused by their bending was excessive.

When the steel rods were placed in the crown one set was wrapped with paper, two sets were thoroughly greased and the fourth set left untreated, but no difference in action was visible between any of them, showing that the bond between the steel and the concrete was broken without causing any undue destruction of the concrete.

The load remained on the arch for about one month without showing any signs of increase in the cracks already visible, although the deflection was not determined. At the end of that time about half of the corner piles were thrown upon the center of the arch without any appreciable change. In unloading, one half of the arch was unloaded first, leaving the other half under its full load, again without any signs of failure. Then the half of the load remaining lying nearer the edge was removed, leaving a load of approximately six tons on the middle quarter of

the floor and again without any signs of failure on the part of the arch.

When it was desired to tear down the arch it was only necessary to take a sledge hammer and knock away the concrete from around the rods at the crown, cut the rods there with a hack saw, then a few blows of the sledge hammer separated the arch into four sections, which were easily removed.

CONCLUSION.

I have attempted to show in the foregoing pages the reasoning pursued in the application of groined arches to concrete floor construction, and the methods used in the designing and testing of the model. I make no claim to originality in the theoretical design, my desire being chiefly to employ the easiest methods to find if there were any serious difficulties to be encountered in this new method of floor construction. The results of the test, while not of the highest scientific accuracy, owing to the limited time available for carrying on this work, were conclusive enough to show that there is great opportunity for the development of floor construction along the lines outlined in this paper.

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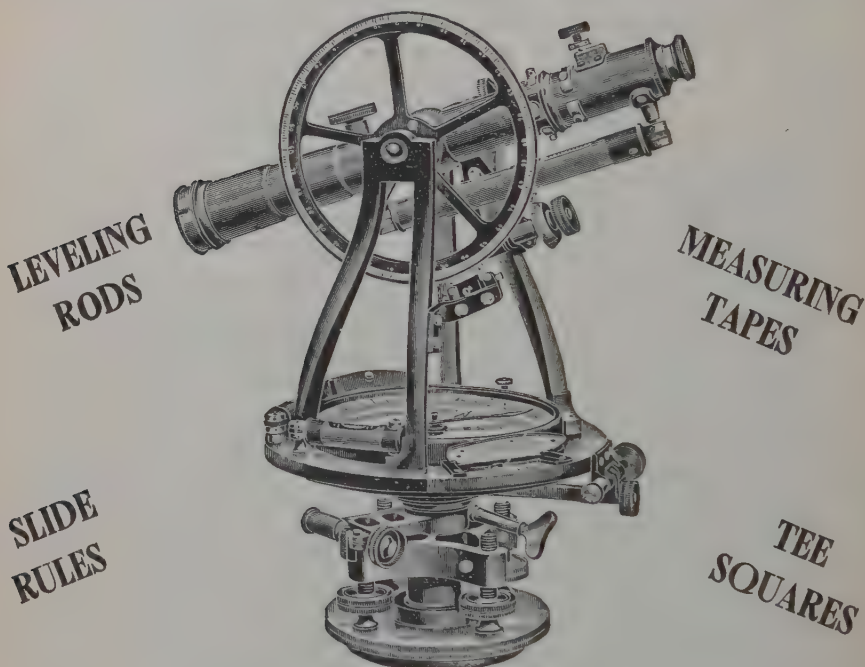
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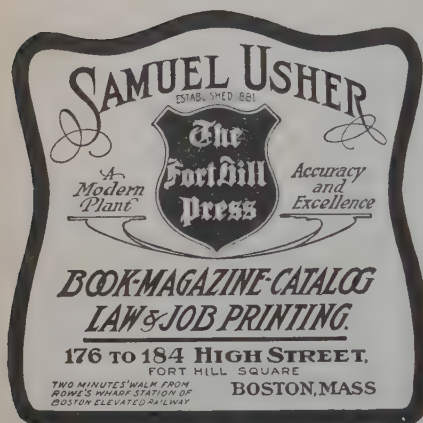
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